FINAL REPORT

"AN INVESTIGATION OF THE LOAD CARRYING CAPACITY OF DRILLED CAST-IN-PLACE CONCRETE PILES BEARING ON COARSE GRANULAR SOILS AND CEMENTED ALLUVIAL FAN DEPOSITS"

By

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and
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Submitted to

The Arizona Highway Department
Phoenix, Arizona 85007

For

Research Project - Arizona HPR-1-10(122)
Drilled Cast-In-Place Concrete Piles

Sponsored by

The Arizona Highway Department
in cooperation with
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Sergent, Hauskins & Beckwith
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16. Abstract

SEVEN LOAD TESTS OF DRILLED PILES WERE PERFORMED ON COARSE GRANULAR SOILS AND 20 TESTS ON CEMENTED ALLUVIAL FAN DEPOSITS. MAXIMUM TEST LOAD WAS 1000 TONS. DETAILED SITE SELECTION AND SITE SOIL INVESTIGATION STUDIES WERE PERFORMED. A SPECIAL TRAILER-MOUNTED PORTABLE LOAD FRAME WAS DESIGNED AND FABRICATED FOR THE PROJECT. BELLED, STRAIGHT, SMALL MULTIPLE BELL, END-BEARING ONLY AND SIDE SHEAR ONLY TESTS WERE PERFORMED. A TELL-TALE INSTRUMENTATION WAS USED.

A COMPARISON OF VARIOUS SETTLEMENT ANALYSIS AND BEARING CAPACITY CALCULATION METHODS WAS MADE. CALCULATIONS FOR THE FINER-GRAINED CEMENTED ALLUVIAL FAN SOILS WERE BASED UPON CONSOLIDATION, DIRECT SHEAR AND FIELD PRESSUREMETER AND PENETRATION TESTS. DESIGN RECOMMENDATIONS AND CONSTRUCTION PROCEDURES ARE PRESENTED.

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February 16, 1973

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Attention: Mr. Gene Morris
Research Engineer

Re: Contract No. 71-30 (Phase II)
Drilled Cast-In-Place Concrete Piles
Project No. HPR-1-9(172) AFE 60013

Gentlemen,

Submitted herewith is a final draft of our report "An Investigation of the Load-Carrying Capacity of Drilled Cast-In-Place Concrete Piles Bearing on Coarse Granular Soils and Cemented Alluvial Fan Deposits" prepared under the referenced contract. The report includes a description of site selection process, soil and geologic conditions at test sites, soil investigation of test sites, testing equipment and testing procedures along with our evaluation of test results, conclusions and recommendations.

The opinions, findings and conclusions contained in this report are those of the authors and are not necessarily those of the Arizona Highway Department and the Federal Highway Administration.

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Contract No. 71-30 (Phase II)
Drilled Cast-In-Place Concrete Piles
Project No. HPR-1-9(172) AFE 60013
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Upon your review, we are looking forward to discussing the report with you.

Respectfully submitted,

Sergent, Hauskins & Beckwith Engineers

By______________________________

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And______________________________

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Job No. E71-272

NOTATION

THE FOLLOWING SYMBOLS ARE USED IN THIS REPORT:

A = COHESION REDUCTION COEFFICIENT
A_B = END AREA OF PILE
A_S = SIDE AREA OF PILE SHAFT
C = SOIL COHESION
D = BASE DIAMETER OF PILE
D = DEPTH FROM GROUND SURFACE TO BASE OF PILE
E = MODULUS OF DEFORMATION OF SOIL
I_W = STRESS INFLUENCE COEFFICIENT
K = LATERAL EARTH PRESSURE COEFFICIENT
N = STANDARD PENETRATION RESISTANCE, BLOWS/FOOT
N_B = BECKER HAMMER DRILL PENETRATION RESISTANCE, BLOWS/FOOT
N_C = BEARING CAPACITY FACTOR
N_Q = BEARING CAPACITY FACTOR
P_F = CREEP PRESSURE DETERMINED BY THE PRESSUREMETER TEST
P_L = LIMIT PRESSURE DETERMINED BY THE PRESSUREMETER TEST
P_O = INITIAL PRESSURE DETERMINED BY THE PRESSUREMETER TEST
P_Z = INITIAL VERTICAL CONFINING STRESS ON SOIL
S = SETTLEMENT OF PILE
s = SOIL SHEAR STRENGTH DEFINED BY DIRECT SHEAR TEST
s_O = SOIL SHEAR STRENGTH DEFINED BY PRESSUREMETER TEST
Q = TOTAL LOAD ON PILE
Q_B = TOTAL ULTIMATE END BEARING RESISTANCE OF PILE
Q_S = TOTAL ULTIMATE SIDE RESISTANCE OF PILE
Q_T = TOTAL ULTIMATE RESISTANCE OF PILE
Q = UNIT BEARING PRESSURE ON BASE OF PILE, BEARING PLATE OR EQUIVALENT PIER

NOTATION

Q_B = ULTIMATE END BEARING PRESSURE ON BASE OF PILE

Q_S = ULTIMATE UNIT SIDE SHEAR ON PILE SHAFT

Q_U = UNCONFINED COMPRESSIVE STRENGTH OF SOIL

U = POISSON'S RATIO

Z = DEPTH FROM GROUND SURFACE TO POINT BEING CONSIDERED

γ = DENSITY OF SOIL

δ = ANGLE OF FRICTION BETWEEN SOIL AND SHAFT OF PILE

ϕ = ANGLE OF INTERNAL FRICTION OF SOIL

CHAPTER I - INTRODUCTION

Scope & Objectives

This report presents the results of an investigation of the load-carrying capacity of drilled cast-in-place concrete piles deriving support from very coarse granular deposits and cemented finer-grained alluvial fan deposits. These types of soils predominate in the heavily populated areas of central and southern Arizona. The major objective of the study was to develop rational and/or empirical methods of predetermining bearing capacities of drilled piling which could be used in routine design. The study is confined to downward axial loading. A secondary objective of the study was to evaluate design details and inspection procedures for local conditions and provide recommendations for routine use.

Summary of Program

Prior to the start of the actual load testing program, a search was made of available, pertinent engineering literature. The reports of previous studies of drilled piling were reviewed in detail. Following this review, a study of existing soil data was made to select criteria for load test sites, which lead to the selection of several available sites for preliminary test borings. As a result of the above studies, three sites were selected which are believed to be as representative as possible of the range of typical soils for the predominance of actual construction sites. One site was selected for "end-bearing" only studies on a coarse granular deposit and two for "end-bearing" and "side shear" studies in finer alluvial fan deposits.

A detailed investigation was then made of each of the three sites to determine the physical properties of the subsurface materials.

Drilled piling were constructed at each site along with the necessary anchor piling for uplift resistance for the load frame. The piling constructed for load testing were as follows:

1. In the coarse granular materials, 7 end-bearing piles. All but one of these were belled to varying diameters.

2. In the finer alluvial fan deposits where both end-bearing and side shear were evaluated, piling were constructed for end-bearing only, side shear only, with multiple small bells (shear collars) and without cleaning loose materials from base. The last series were used for evaluation of the effectiveness of piling which are "machine cleaned" only.

For the performance of the load tests, a self-contained, hydraulically operated load frame was designed and assembled. The frame capacity is 1000 tons. A "telltale" instrumentation system was developed for use in side shear load transfer measurements.

The tests were evaluated relative to various methods for the predetermination of ultimate bearing capacity and estimation of settlements. Recommended design procedures, construction and design details and inspection procedures were developed.

Soil Conditions Involved in Study

Much of Phoenix and the Salt River Valley are underlain by very coarse granular deposits consisting of mixtures of sand, gravel and cobbles. Similar deposits are present at many

OTHER AREAS IN THE LOWER VALLEYS OF CENTRAL AND SOUTHERN ARIZONA. THESE SOILS WERE DEPOSITED BY HIGH GRADIENT DISCHARGES OF THE SALT RIVER AND OTHER DRAINAGES. THIS SOIL TYPE IS HEREAFTER TERMED SGC IN THIS REPORT. THE SGC SOILS ARE FAR TOO COARSE TO ENABLE EVALUATION OF RELATIVE DENSITY OR COMPRESSIBILITY BY CONVENTIONAL PENETRATION AND LABORATORY TESTS COMMONLY USED FOR SANDS. THEIR COARSE NATURE ALSO MAKES IT EXTREMELY DIFFICULT AND COSTLY TO PERFORM IN-PLACE DENSITIES TO COMPARE WITH LABORATORY MAXIMUM AND MINIMUM DENSITIES. ONE OF THE LOAD TEST SITES IN THIS INVESTIGATION INVOLVED THE SGC DEPOSIT.

ABOUT 30 PERCENT OF THE LAND AREA IN THE ARID PORTION OF THE SOUTHWEST IS UNDERLAIN BY ALLUVIAL FAN DEPOSITS. THESE SOILS TYPICALLY CONSIST OF HIGHLY STRATIFIED SAND-SILT-CLAY MIXTURES. THEY ARE DEPOSITED BY SHEET FLOODS AND DISCHARGES OF SMALL INTERMITTENTLY FLOWING DRAINAGES. IN THIS SEDIMENTATION PROCESS, A LAYER OF SOIL DRIES OUT PRIOR TO DEPOSITION OF AN OVERLYING LAYER. IN WELL DRAINED AREAS, SOIL MOISTURE CONTENTS STABILIZE AT VERY LOW VALUES (WELL BELOW THE PLASTIC LIMIT) IN THE ARID ENVIRONMENT WHERE ANNUAL EVAPORATION EXCEEDS RAINFALL BY 60 INCHES OR MORE. IN SOME INSTANCES, DEPENDING ON THE NATURE OF THE PARENT MATERIAL AND EXACT MANNER OF DEPOSITION, ALLUVIAL FANS PRODUCE LOOSE MOISTURE SENSITIVE "COLLAPSING" SOIL DEPOSITS. IN MANY OTHER CASES, ALLUVIAL FANS RESULT IN LIME CEMENTED DEPOSITS WHICH ARE FIRM TO HARD IN CONSISTENCY AND NOT GREATLY WEAKENED BY MOISTURE INCREASES. THIS WIDESPREAD GENERAL CATEGORY OF DEPOSIT IS THE SECOND TYPE OF SOIL INVESTIGATED IN THIS STUDY AND IS HEREAFTER TERMED CAF (CEMENTED ALLUVIAL FAN) SOILS IN THIS REPORT.

THESE RELATIVELY DRY ALLUVIAL FAN SOILS ARE USUALLY FISSURED AND FRACTURED TO SOME DEGREE, POSSESS A FRIABLE TEXTURE AND often contain gravel particles and calcareous concretions and nodules. Because of these characteristics, sampling normally cannot be accomplished with thin walled or double barreled tube samplers and often, block samples cannot be efficiently cut and trimmed. The use of thick walled open-end drive samplers lined with metal or plastic rings is necessary for efficient sampling of CAF soils and usually the samples cannot be extruded for triaxial or unconfined compression testing. Thus, the direct shear test has been widely adopted for Arizona soil conditions because testing apparatus will receive the liner rings with soil specimens as secured in the field. Because of the sample disturbance induced by the high driving energy involved in open-end drive sampling, the frequent presence of scattered coarser soil particles or concretions and the theoretical limitations of the test, the validity of direct shear test data obtained on CAF soils as it applies to analysis of pile capacities has been considered highly questionable in many cases.

The Use of Drilled Cast-In-Place Concrete Piles in Arizona

With the development of modern foundation drilling equipment since World War II, the use of high capacity drilled cast-in-place concrete piles has gained wide and steadily increasing use in many areas of the world. Development of the use of large diameter piles in the United States has been reviewed by White (1)*.

*Numbers in parenthesis correspond to references listed in Appendix A.

The soil conditions prevalent in Arizona lend themselves to the efficient use of drilled piling. In many areas, a surface layer of looser moisture sensitive alluvial fan materials, typically 8 to 25 feet in depth, overlie SGC or CAF soils. Belled piles bearing near the surface of the SGC or CAF soils have usually been employed for these conditions. In cases where CAF soils are near the surface, straight piles penetrating these deposits have often been used.

For the typical conditions previously discussed, excavations for drilled piles can be made without the use of casing and without significant caving and sloughing occurring. Because of these very favorable conditions for installation and the efficiency of available drilling equipment, drilled piles have proven considerably more economical than driven piling for typical Arizona conditions. Thus, the majority of heavily loaded building structures in Phoenix and Tucson constructed in the past 12 years are supported on drilled piles.

Drilled piles have proven to be particularly advantageous because the high concentrated loads involved for many projects can be supported on single piles. Building projects in Arizona have involved loads up to about 2000 tons imposed on single piles. Future elevated freeway structures in Arizona may involve long-term concentrated loads of 5000 tons or more. Drilled piles have also proven efficient where high upward or lateral loads are imposed and have been widely used in Arizona for electrical transmission towers and similar structures.

The Problem of Predetermining Pile Capacities

No dynamic driving record is developed during construction of drilled piles and load tests are very costly for most projects due to the high capacities involved. Thus, it is necessary,

IN MOST CASES, TO PREDETERMINE CAPACITIES BY RATIONAL OR EMPIRICAL METHODS FOR EVALUATION OF SETTLEMENT UNDER WORKING LOADS AND ULTIMATE CAPACITY.

THE PROBLEM OF BEARING CAPACITY PREDETERMINATION IS PARTICULARLY DIFFICULT FOR BOTH SGC AND CAF SOILS BECAUSE OF PREVIOUSLY DISCUSSED PROBLEMS IN EVALUATING ENGINEERING PROPERTIES BY FIELD AND LABORATORY TESTS. WIDE HORIZONTAL AND VERTICAL VARIATIONS OF THE PROPERTIES OF THESE SOILS, EVEN AT THE MOST UNIFORM SITES, ALSO COMPOUNDS THE PROBLEM OF PILE CAPACITY PREDETERMINATION. BECAUSE OF THIS FACTOR, IT WAS JUDGED NECESSARY TO TEST AS MANY PILES IN THESE MATERIALS AS POSSIBLE. FOR THIS REASON, A RELATIVELY SIMPLE AND INEXPENSIVE INSTRUMENTATION SYSTEM WAS SELECTED.

AS PREVIOUSLY STATED, THE COARSE NATURE OF THE SGC DOES NOT ALLOW THE PERFORMANCE OF MEANINGFUL PENETRATION OR SOIL MECHANICS TESTS. DESIGN PRACTICE FOR BELLED PILES BEARING ON THE SGC HAS BEEN TO ASSIGN SAFE SOIL BEARING PRESSURES BASED ON CLASSIFICATION AND PRECEDENT WITH PROJECTS BOTH IN ARIZONA AND ELSEWHERE. SAFE SOIL BEARING PRESSURES RANGING FROM ABOUT 8000 TO 20,000 PSF HAVE BEEN USED FOR VARIOUS PROJECTS DEPENDING UPON TOTAL CONCENTRATED LOADS AND EXTENT OF SUBSURFACE EXPLORATION PERFORMED. FOR THE BEARING PRESSURES EMPLOYED ON THE BASIS OF PRECEDENT AND THE DEPTH AND WIDTH OF TYPICAL BELLED PILES, EXTREMELY HIGH FACTORS OF SAFETY ARE COMPUTED BY THE SEMIEMPIRICAL TERZAGHI EQUATION FOR END-BEARING (2) USING SHEAR STRENGTHS FOR SGC ASSIGNED FROM VISUAL CLASSIFICATION. THUS, THE PRACTICAL PROBLEM OF DESIGN OF FOUNDATIONS BEARING ON THE SGC IS ONE OF ASSESSING SETTLEMENTS UNDER WORKING LOADS. SEVEN 1000 TON LOAD TESTS WERE PERFORMED IN THIS STUDY ON END-BEARING PILES OF VARIOUS WIDTHS BEARING ON SGC. THE MAJOR OBJECT OF THESE TESTS WAS TO INVESTIGATE THE RELATIONSHIP BETWEEN WIDTH OF LOADED AREA AND SETTLEMENT FOR VARIOUS BEARING PRESSURES.

The recently developed Becker Hammer Drill has provided an efficient method of exploration of the SGC. This method, involving a dynamically advanced double walled drive pipe and removal of cuttings with compressed air by a reverse circulation process, enables relatively accurate classification of coarse granular deposits. Classification in conjunction with the dynamic driving record developed provided a tool for comparison of the SGC load test site and other sites, and assessing the overall uniformity of the deposit. Prior to the advent of the Becker Drill, the cable-tool percussion drill rig was the only means of obtaining this data. From a practical standpoint, the rate of drilling in the SGC with a churn drill is too slow for major projects, even though the cost in some cases does not greatly exceed that for Becker drilling.

In local practice, various rational methods for computing the ultimate capacity of drilled piling based upon the summation of side shear for various layers and end resistance have been used for the analysis of both belled and straight piles bearing on CAF soils. A major difficulty in this procedure is in the sampling and performance of shear tests as previously discussed. Various rational settlement analysis procedures also have been employed in design analysis. The effect of sample disturbance on consolidation tests of CAF soils which are heavily overconsolidated by desiccation, introduces large uncertainties as to the validity of settlement computations.

Previous Studies

Although, to our knowledge, no comprehensive studies of drilled pile capacities in the types of soils studied in this investigation have been performed to date, a large body of data on drilled piles has been reported in engineering literature. Much of this data was not only pertinent to the design and evaluation of tests performed in this study, but is applicable to the design of drilled piling in other soil types which occur in Arizona.

The Center for Highway Research at the University of Texas at Austin has performed detailed studies of various aspects of drilled pile design which have been presented in a series of nine research reports published between April, 1968 and December, 1970. In one of these reports, O'Neill and Reese (3) present a comprehensive review of both the state of the art of empirical and rational methods for the analysis of pile capacities and settlements and of previous studies on drilled piles.

A large number of studies have been made on stiff, fissured clays which were generally saturated or at high moisture contents. A number of these soils are similar to those which occur in some areas of northern Arizona. Among the locally significant studies in this category are those of Skempton (4); Whitaker and Cooke (5); and Burland, Butler and Dunican (6) on London clay; Reese and Hudson (7) and O'Neill and Reese (3) on Texas soils; Mohan and Chandra (8) on Indian Black Cotton soils; Woodward, Lundgren and Boitano (9) in California; and Watt, Kurfurst and Zeman (10) in Canada.

Several load test programs have been performed on shales similar to one of the types that occur in the Colorado Plateau area of northeast Arizona. Among these are data presented by Reese, Hudson and Vijayvergiya (11) and U. S. Army Engineers (12) on Texas sites; Van Doren, et al (13) on a Kansas site; and Matich and Kozicki (14) on a Canadian site. A South Dakota Department of Highways study by Bump, et al (15) involved the

Pierre shale which is very similar to the widespread Chinle shale in northeastern Arizona. In most of these studies, however, moisture contents were considerably higher than at the typical Arizona site.

Factors involved in the design of drilled piles in sand are presented by Vesic (16). Martins (17) presents tests on drilled piles below the water table in predominantly sandy soils. Barker and Reese (18) have reviewed various research on load test data on drilled piling installed below the water table with bentonite slurry. This method may be economically competitive in Arizona at certain sites on the Colorado River and major drainages in northern Arizona.

CHAPTER II - SELECTION OF TEST SITES

General Site Selection Study

At the beginning of this study, the two basic types of soils to be investigated were recognized. It was concluded that a site for end-bearing load tests on the SGC and two load test sites on typical CAF soils would reasonably cover the majority of soils in Arizona not already investigated in previous research.

A detailed study of soil conditions prevailing in the Salt River Valley, the Tucson area and other valleys in central, southern and western Arizona was made to enable selection of as representative a group of sites as possible. About 2000 previous soils investigations made by this firm, including the preliminary investigation for the Papago East Freeway in Phoenix (19) and the I-710 extension in Tucson (20), were reviewed. Previous surface soils mapping of the Salt River Valley by the U.S. Bureau of Soils and Chemistry (21) and studies of the valley by the U. S. Geological Survey (22, 23, 24) also were reviewed.

General Criteria for Site Selection

Beyond the basic criteria of selection of sites that as well as possible represented the range of soils involved, several other general factors were considered in site selection. It was desired to locate the sites in the Phoenix area to minimize travel time. It was also considered preferable that the sites be on acquired right-of-way owned by the Arizona Highway Department to minimize administrative problems and to enable fencing for security in that long-term storage of equipment would be involved in the program.

It was further considered desirable that the sites be located within the area of freeway alignments where construction of elevated viaducts involving a large number of high capacity foundations is programmed for the relatively near future. It was thought that this might enable the development of supplementary information by measuring settlements of actual foundations during construction and operation of the freeways.

Special Criteria - SGC Site

Several special criteria for the selection of the SGC site were established as follows:

1. The site should be located in a uniform area of the SGC where the deposit is uncemented and thus represents the weakest general condition. A minority of the upper soils in the SGC deposit possess varying degrees of cementation. The deposit should be free of clay and sand layers beneath the test site.

2. Settlement of foundations on granular soils tend to decrease somewhat with increasing depth, all other factors being equal. Thus, the site should be in a relatively shallow area in order to test the most critical condition of depth commonly involved in drilled foundation design for local conditions.

3. The surface soils over the SGC should be strong enough to efficiently develop the necessary uplift capacity of belled reaction piles founded on the surface of the SGC.

Special Criteria - CAF Sites

The CAF soils under study embrace a wide variety of soil classifications with silty clays, sandy clays and clayey sands predominating (Unified Soil Classification CL and SC). Lesser amounts of highly plastic clay and sandy clays silty

SANDS, CLAYEY SILTS AND SANDY SILTS (CH, SM AND ML) OCCUR. WIDE VARIATIONS IN THE DEGREE OF CEMENTATION ALSO OCCUR WITH CALCAREOUS CONCRETIONS AND MINOR FRACTIONS OF GRAVEL BEING PRESENT IN MANY CASES. VARIATIONS IN THE GRANULAR FRACTION OF SOILS AND CHARACTER OF THE CEMENTATION PRODUCE WIDE DIFFERENCES IN THE SURFACE TEXTURE OF THE FACE OF DRILLED PILE EXCAVATIONS. IN VIEW OF THE FACT THAT SURFACE TEXTURE HAS BEEN SHOWN TO BE AN IMPORTANT FACTOR INFLUENCING THE DEGREE TO WHICH SOIL SHEAR STRENGTH IS MOBILIZED IN SIDE SHEAR, THIS WAS CONSIDERED IN SITE SELECTION. THE DEGREE AND CHARACTER OF CEMENTATION SUBSTANTIALLY INFLUENCES STRENGTH AND WAS THUS CONSIDERED.

THE INTENSITY, ORIENTATION AND SURFACE TEXTURE OF JOINTING AND FISSURING ARE ALSO IMPORTANT FACTORS IN SOIL SHEAR STRENGTH RELATIVE TO PILE CAPACITIES AND WERE CONSIDERED. CAF SOILS ARE TYPICALLY OVERCONSOLIDATED BY DESICCATION TO A CONSIDERABLE DEGREE AND PROBABLY POSSESS A RELATIVELY HIGH STATE OF HORIZONTAL STRESS IN SITU. IT WAS RECOGNIZED IN SITE SELECTION, THAT OVERCONSOLIDATION IS PROBABLY INTERRELATED WITH FISSURING AND JOINTING AND THAT STRESS RELIEF UPON EXCAVATION OF FOUNDATIONS MIGHT SIGNIFICANTLY AFFECT CAPACITY.

FINAL SITE SELECTION

GENERAL STUDIES REVEALED AN SGC AREA ALONG THE PROPOSED PAPAGO EAST FREEWAY IN PHOENIX WHICH APPEARED TO MEET CRITERIA AND AREAS ALONG PAPAGO WEST AND SUPERSTITION FREEWAY ALIGNMENTS WHICH APPEARED SUITABLE FOR THE TWO CAF SITES. THE AREAS WERE ON ACQUIRED RIGHT-OF-WAY AND AT CONVENIENT LOCATIONS. EXTENSIVE EXPLORATORY DRILLING WAS PERFORMED IN THESE AREAS TO VERIFY CONDITIONS INDICATED BY THE GENERAL STUDY. AS A RESULT, 3 LOAD TEST SITES WERE SELECTED

IN THESE AREAS DESIGNATED A THROUGH C. A VICINITY MAP, FIGURE 9*, SHOWS TEST SITE LOCATIONS. FIGURES 10 AND 11 SHOW EXACT TEST SITE LOCATIONS ON AERIAL PHOTOGRAPHS.

*TABLES AND FIGURES ARE GIVEN IN APPENDIX B.

CHAPTER III - SOIL INVESTIGATIONS OF TEST SITES

General

Detailed auger drilling, penetration testing and open-end drive sampling were performed at each site in the CAF soils by the procedures normally used in subsurface exploration in local practice. The purpose of this type of drilling at Site A was to provide data on the surface CAF soils for the design of reaction piles and comparative information on the various field and laboratory tests. In addition, both large diameter test holes and Becker Hammer Drill holes were made into the SGC soils at Site A. Detailed laboratory testing was performed on the recovered samples.

Several in situ testing methods were considered for use in the hope that higher quality data on strength and compressibility of the CAF soils could be obtained than normally achieved in the laboratory. The Pressuremeter, a balloon-like load cell that is employed to perform load tests in small diameter drill holes, was selected for use. Several studies using this device have shown promising results in stiff soils and soft rock (25, 26, 27, 28, 29).

Several other in situ testing methods were considered for use for investigating the shear strength of CAF soils. Mechanical methods such as the Handy Iowa State device (30) which involve expansion of plates against the borehole walls do not appear applicable because of point loading on the rough sides of the holes created in most CAF soils. A large diameter device of this type studied by Campbell and Hudson (31) did not perform well. A relatively high capacity vane shear apparatus was developed for investigations of the 1964 Alaska earthquake (32). However, its capacity is only 5000 psf (not enough to fail many CAF soils) and point loading of vanes on gravel particles and calcareous concretions would prevent its use in many cases. It was also thought that point loading on coarse particles would create serious problems with static cone penetration tests.

Subsurface Exploration

Five large diameter borings were drilled around the perimeter of Site A to about 15 feet into the SGC. The purpose of these borings was to investigate the deposit for layers of sand or clay. These borings were drilled with a Texoma 500-35 drill rig and 30 inch diameter noncontinuous flight auger.

Test borings were drilled at each corner of each test site with 6 5/8 inch O.D., $3\frac{1}{4}$ inch I.D. hollow stem auger and a CME-55 drill rig. The borings refused on the SGC at between 13 and 16 feet at Site A. The auger borings were drilled to about 40 feet at Site B and 25 to 30 feet at Site C.

At each boring location, a standard penetration test was performed at $2\frac{1}{2}$ foot intervals in an auger boring by the ASTM D1586-67 procedure (33). About 4 feet away, a second auger boring was drilled and open-end drive sampling performed at $2\frac{1}{2}$ foot intervals. Three inch O.D. samplers were used lined with 2.42 inch I.D. brass rings. The sampler drive shoe had the same I.D. as the brass liner rings (zero relief). The open-end drive samplers were driven with a 140 pound, 30 inch free fall drop hammer in the manner outlined in ASTM D1586-67. Blows required to advance the 2 inch O.D. standard penetration samplers and the 3 inch O.D. open-end drive samplers are recorded on the boring logs in 2 inch increments in order to provide information on the degree of stratification and the presence of thin, strongly cemented layers and scattered gravel and calcareous concretions.

At a location about 4 feet from the auger borings, continuous penetration tests were performed by driving a 2 inch O.D. blunt nosed penetrometer. The penetrometer consists of a 2 inch O.D. heavy cut washer slipped over a nipple which is attached to 1 5/8 inch O.D. drill rods (A-rod). Penetration values are recorded as the number of blows of a 140 pound, 30 inch free fall hammer required to advance the penetrometer in one foot increments or less. This "bullnose" penetration test has been widely used in local practice.

In order to provide detailed information on the nature of the SGC beneath Site A, borings were advanced with a Becker Hammer Drill immediately adjacent to each test pile after the load tests were complete. The principal features of the rig are shown on Figure 12. The Becker Hammer Drill advances a double walled drive pipe with a Link Belt pile driving hammer rated at 8000 foot-pounds energy per blow. Cuttings are removed by compressed air by a reverse circulation process. The drive pipe used in the test drilling was $5\frac{1}{2}$ inch O.D. x $3\frac{1}{4}$ inch I.D. and employed an expendable drill bit of slightly larger diameter than the O.D. of the drive pipe. Hammer blows required to advance the drive pipe were recorded in 6 inch increments and representative samples of cuttings were obtained.

Logs of test borings are presented in Appendix C. Figures 13, 14 and 15 show the location of borings at the test sites.

Laboratory Tests

Moisture content determinations were made on all samples recovered in standard penetration testing and open-end drive sampling while dry densities were determined for the open-end drive samples. Results of these tests are given on the boring logs.

Grain-size analysis and Atterberg Limits tests were performed on selected samples of the various soils involved.

One dimensional consolidation tests were performed on selected samples of the CAF soils from each site. "Floating ring" apparatus designed to receive one inch high 2.5 inch O.D. brass liner rings with soil specimens as secured in the field were used in the tests. Procedures were generally those outlined in ASTM D2435-70 (33). The specimens were consolidated to loads approximately equal to the preconsolidation pressure and rebounded with the tests then being completed in the normal sequence. In the partially saturated soils involved, each increment of load was maintained until the rate of deformation was equal or less than 0.0001 inch per hour.

Direct shear tests were run on all open-end drive samples using an apparatus of the strain-control type. Shearing forces were applied at a rate deformation of approximately 0.05 inches per minute. The machine is designed to receive one of the one inch high 2.42 inch diameter specimens obtained by sampling. Generally, each sample was sheared under a normal load equivalent to the effective overburden pressure at the point of sampling. In some instances, samples are sheared at several normal loads. The plates of the direct shear device were separated 0.04 inch during the tests.

Unconfined compression tests were performed on a few block samples obtained in an anchor pile excavation at Site B. Because of the fissured, friable nature of the CAF soils block samples could not be obtained at Sites A and C.

Results of the laboratory tests are presented in Appendix C.

Pressuremeter Tests

In situ tests on the walls of small diameter boreholes were made in the CAF soils at all 3 sites with a pressuremeter. The boring locations were about 4 feet from the auger boring and continuous penetration test locations. The pressuremeter equipment now in use, shown schematically in Figure 16, was developed in the 1950s by Menard (34) in France and gained wide use in Europe since. The device became available in the United States in the mid 1960s.

The pressuremeter apparatus consists of a steel probe with rubber membranes forming 3 independent pressure cells which is lowered into testing position in the borehole. The cells are connected by plastic lines to a combined volumeter-manometer control apparatus at the surface. Pressure-deformation relationships are determined by observing the volume of water injected into the central cell during incremental pressure increases. The outer guard cells expand under an equal pressure and reduce the end effects on the central measuring cell. Detailed descriptions of the pressuremeter and its application have been presented by a number of authors (25, 26, 27, 28, 29, 34, 35).

A typical volumetric strain versus pressure curve for the stiffer CAF soils encountered in the study are shown in Figure 16. Several basic quantities are obtained in the test. The "initial" pressure, Po, is the beginning of the relatively elastic portion of the curve. In the portion of the curve up to Po, the natural horizontal stresses in the soil are restored. At pressures beyond the relatively linear phase, the rate of volume change increases rapidly until the limit pressure, Pl, which is considered to define failure, is reached. Also plotted on Figure 16 is the creep curve which shows the tendency of the material to deform with time. The creep pressure, Pf, is the pressure at which the creep curve takes a sharp upward break. This point generally agrees closely with the upper limit of the linear portion of the pressure-volume curve. The modulus of deformation, E, is derived from the slope of the curve between Po and Pf.

Pressuremeter probes designed to test NX (2.98 inch diameter) holes were used in this investigation. The initial uninflated diameter of the probe is slightly smaller than NX hole size and inflates to about twice the initial diameter. In order to perform tests in the stiff CAF soils, it was necessary to drill holes with the diameter varying between about 2.75 to 3.00 inches. Where the hole diameter was larger, the ends of the membranes tended to rupture before Pl could be defined.

Considerable difficulty was experienced in drilling holes within acceptable tolerance of diameter in the friable, fissured soils involved. This represents a basic limitation of application of pressuremeter testing to some strongly cemented, fissured CAF soils. Initially, it was attempted to drill pressuremeter holes with 2.50 inch diameter continuous flight auger. This auger proved insufficiently rigid to effectively drill holes except in about the upper 10 feet at Site B. Tricone and bicone gear bits of various diameters and a special fabricated drag bit, 2.25 inches in diameter, were used in initial attempts in drilling. Compressed air drilling methods were used with these drilling tools. The use of tricone gear bits, 2.94 inches in diameter, resulted in holes far larger than required for testing. After considerable experimentation, a system of drilling with a 2.37 inch diameter gear bit or 2.25 inch drag bit then reaming the holes with a specially fabricated 2.75 inch pipe with sawcat teeth was developed. This technique proved very successful in the CAF soils at Sites A and B and a large number of tests were efficiently performed after the initial development of the system. Extreme difficulty was experienced at Site C due to the degree of fissuring and only 4 satisfactory tests were accomplished with considerable effort.

Results of the pressuremeter tests are presented on Table 6 and shown graphically on Figure 76.

CHAPTER IV - GENERAL GEOLOGY & CHARACTER OF SOILS INVESTIGATED

GENERAL GEOLOGY & SOIL CHARACTERISTICS

THE GEOLOGY OF THE GREATER PHOENIX METROPOLITAN AREA IS TYPICAL OF THE LOWER VALLEYS OF ARIZONA WHERE SGC AND CAF SOILS ARE INVOLVED IN CONSTRUCTION.

THE SALT RIVER EMERGES FROM A NARROW CANYON EAST OF PHOENIX INTO THE BROAD SALT RIVER VALLEY. THE HIGH GRADIENT DISCHARGES OF THIS RIVER HAVE RESULTED IN MASSIVE DEPOSITS OF SGC. THE AGUA FRIA RIVER, GILA RIVER, NEW RIVER, CAVE CREEK, SKUNK CREEK AND QUEEN CREEK HAVE CREATED SIMILAR DEPOSITS OF LESSER EXTENT. THE GEOLOGY OF THE SALT RIVER VALLEY IS DESCRIBED BY LEE (22) AND MCDONALD, ET AL (23) WHILE THE GEOLOGY OF THE WESTERN PART OF THE VALLEY IS DISCUSSED BY STULIK AND TWENTER (24). THE SGC SOILS AND MUCH OF THE CAF ALLUVIUM IN THE PHOENIX AREA WERE DEPOSITED DURING THE TERTIARY PERIOD. THE DEPOSITION WAS CAUSED BY UPLIFT OF THE HIGH PLATEAU COUNTRY NORTH OF THE MOGOLLON RIM AND A CORRESPONDING WIDELY EXTENDED SUBSIDENCE OF THE AREA TO THE SOUTH AND WEST RESULTING IN DEEP EROSION OF THE HIGHLAND COUNTRY AND RAPID FILLING OF THE VALLEY AREAS. A MORE RECENT EROSIONAL PHASE OF THE SALT RIVER CHANNEL ASSOCIATED WITH A PERIOD OF DRIER CLIMATE IS DESCRIBED BY LEE (22) WHO MAPPED SGC TERRACES IN THE MESA AREA ABOUT 50 FEET HIGHER THAN THE PRESENT CHANNEL. SIMILAR TERRACE LEVELS PRESENT IN THE PHOENIX AREA ARE OBSCURED BY THE OVERLYING LAYER OF ALLUVIAL FAN DEPOSITS. WELL LOGS INDICATE THAT THE SGC DEPOSITS EXTEND TO SEVERAL HUNDREDS OF FEET IN MANY AREAS OF THE VALLEY.

THE SGC SOILS ARE EXPOSED AT THE SURFACE IN THE STREAM CHANNELS AND ALSO ARE EXPOSED IN NUMEROUS MATERIALS PITS and roadway cuts in the Phoenix area. They are overlain by a thin layer of loose silty sands and sandy silts over much of the floodplains of the various major drainages. Adjacent to the floodplains, the SGC are overlain by coalescing alluvial fan deposits sedimented by sheet floods and intermittent flows from small drainages out of the surrounding mountains. They consist predominantly of silty clays, sandy clays and clayey sands (Unified Soil Classification SC and CL) with lesser amounts of highly plastic sandy clays, silty sands, sandy silts and relatively clean sands (CH, SM, ML, SP, SW, SW-SM, SP-SM).

Because these soils are formed by a "flash flood" type of deposition, each layer dries out prior to the deposition of the overlying layer. When the overlying layer is deposited, only the surface of the existing soils are rewetted. Thus, the mass of these deposits have never been consolidated in a saturated state under overburden pressures. Where low density soils, particularly silty sands, sandy silts and clayey sands of relatively low plasticity are involved, this depositional process often results in extremely moisture sensitive "collapsing" soils. Dudley (36) presents a general review of this phenomenon. Parent materials in the area of the study are predominantly granitics, schists and volcanics. A very high incidence of severely collapsing soils is usually involved with granitic parent materials. These soils which are severely weakened by moisture increase and often relatively high in permeability are, of course, not considered suitable for drilled foundations supporting heavier loads.

Considerable thicknesses of "moderately" moisture sensitive soils also occur which, although firmer than the very low density deposits, are weakened by moisture increases to an extent that they are not generally thought to be safe for support of heavy foundation loads. Usually, at least 1 or

2 feet of moisture sensitive alluvial fan soils are present at the surface. These deposits seldom extend below about 25 feet.

A great proportion of alluvial fan soils are lime cemented to varying degrees and are not greatly affected by moisture increases. They usually contain high enough clay contents so they are relatively low in permeability. Thus, they are, in most cases, unlikely to become saturated to depths of more than a few feet in the environmental conditions involved. These soils occur at or near the surface in many areas and are nearly always shallow enough to allow efficient construction of drilled cast-in-place piles for most projects. These highly stratified and cross-bedded soils are generally fractured and fissured to some degree and contain scattered gravel and calcareous concretions in many cases. Except where heavy irrigation has taken place, moisture contents are at the very low values associated with well drained areas in the dry climate of the Sonoran Desert (average annual rainfall in Phoenix is about 7 inches). The infiltration of surface waters into these soils and subsequent drying and precipitation of salts, apparently has created the cementation present. These relatively stable cemented alluvial fan deposits have been termed CAF soils for purposes of this report. All of the soils at Sites B and C below 1 or 2 feet fall into this category.

Character of the SGC

The range of gradation of typical SGC samples are shown in Figure 19. Also tabulated in Figure 19 are particle shape and approximate percentage of rock types making up each size range for a typical sample of the SGC. The SGC consists predominantly of sandy gravel and cobbles with a small amount of silt and generally classifies GP in the Unified Soil Classification System. It generally contains some particles up to about 12 inches and sometimes contains scattered boulders up to about 24 inches. As indicated, the SGC contains a very high percentage of quartzite, chert and other very hard particles. This is reflected by very high wear on drilling tools in both foundation drilling and exploratory drilling into the deposit.

The approximate extent of the area within the Salt River Valley where, because of a relatively shallow contact and the character of the overlying alluvial deposits, it is economical for heavier structures to extend drilled foundations to the SGC is shown on Figure 9.

Local Discontinuities in the SGC

Both exposure on the walls of gravel pits and extensive test drilling indicate that the SGC is relatively uniform for a river channel deposit. However, local discontinuities occur which are significant from the engineering standpoint. Loose river deposited clean sand layers overlie the SGC at isolated locations. Because these layers sometimes contain scattered cobbles they are hard to distinguish from the SGC in some instances in exploratory auger drilling. Caving in these sand zones sometimes creates severe difficulties in advancing drilled foundations or other excavations to the contact of the SGC. In a small number of cases, loose sand or soft clay layers have been encountered within the SGC. Because of the erratic distribution of these features, careful subsurface exploration is necessary to insure uniformity of the SGC at sites of heavier structures.

In general, in the Phoenix area, about the upper 30 feet of SGC is uncemented or very weakly cemented. However, in some areas, the upper few feet of the SGC are strongly cemented;

apparently due to the leaching and precipitation of carbonates from overlying soils. These upper cemented zones are distributed at scattered locations around the Salt River Valley. Some of the more cemented zones in the SGC display the characteristics of lean concrete. The SGC below about 30 feet is generally moderately cemented with clay and other agents and contains some strongly cemented layers. The cementation in some cases may be associated in part with the older terrace levels discussed earlier.

Soil Profile - Site A

Uncemented or weakly cemented silty clays and sandy clays extend to about 7 feet. Moderately to strongly lime cemented clayey sands and sandy clays underlie these soils and extend to about 11 to 13½ feet and in turn rest on moderately cemented clayey gravels. SGC was encountered at between about 14½ and 16 feet. The SGC is uncemented and relatively uniform except for clean, fine to medium sand encountered at between 17½ and 19 feet and 22 to 23½ feet at TPA-7.

Soil moisture contents were very low throughout the extent of the borings.

Soil Profile - Site B

Moderately firm silty clays with a few stratifications of clayey sand and silty sand extended from the surface to about 19 feet. These soils are, in general, weakly cemented. They are underlain by hard, strongly cemented, highly plastic clays, silty clays and sandy silts which extended to about 27 feet. These soils in turn rest on firm to very firm, moderately to strongly cemented silty clays, sandy silts and clayey sands.

Soil moisture contents were variable, ranging from well below to near the plastic limit. Moisture contents appeared to be elevated somewhat above those normally found in well drained desert areas. The site had been under cultivation in the past and irrigation during that period probably created elevated moisture contents. The soils are friable and contain a moderate amount of fissuring or jointing, generally spaced at 6 inches or more. As the boring logs indicate, the soils are highly stratified with a considerable degree of lateral variation being present across the small test site. A relatively smooth surface texture was produced on the walls of drilled pile excavations; particularly in the upper 19 feet.

Soil Profile - Site C

A layer of moderately firm, weakly to moderately cemented silty clay 2 or 3 feet in thickness is present at the surface. This layer is underlain by highly stratified clayey sands and sandy clays of medium to high plasticity which extend to 30 feet; the full depth of investigation. These soils are generally moderately to strongly lime cemented and contain varying amounts of gravel. A thin, very strongly cemented zone about 1 to 3 feet in thickness was encountered in the range of 6 to 9 feet in depth in the various test holes.

Soil moisture contents were generally well below the plastic limit with moisture conditions being typical for the desert environment. The soils are intensely fissured with fissure spacing being about 1 inch. However, the mass of soil appeared to be weakened by fissuring much less in a vertical than a horizontal direction. Stronger, less weakened horizontal layers occurred at various intervals throughout the depth of piles. However, by actual digging in the wall of the drilled shaft with a prospector's pick, a general

CONDITION OF GREATER VERTICAL THAN HORIZONTAL STRENGTH WAS NOTED AT ALL DEPTHS. THE SURFACE TEXTURE OF THE FISSURING IS ROUGH. THIS WAS SPECIFICALLY NOTED AT THE TIME SHEAR COLLARS WERE CUT IN THE WALLS OF THE DRILLED SHAFTS BY HAND METHODS. OWING TO THE SAND AND GRAVEL FRACTION OF THE SOIL, AND THE PRESENCE OF FISSURING AND CALCAREOUS CONCRETIONS, A ROUGH SURFACE TEXTURE WAS PRODUCED ON THE WALLS OF DRILLED FOUNDATION EXCAVATIONS.

CHAPTER V - LOAD FRAME, INSTRUMENTATION SYSTEM & LOAD TEST PROCEDURES

DESIGN OF LOAD FRAME & HYDRAULIC JACKING SYSTEM

PREVIOUS STUDIES BY WHITAKER AND COOKE (5) AND O'NEILL AND REESE (3) INDICATE THAT TESTS OF ABOUT 1000 TONS ARE THE PRACTICAL LIMIT FOR A PROGRAM OF THIS TYPE BECAUSE OF THE COST AND SIZE OF FRAMES NECESSARY TO TRANSFER LOADS FROM THE REACTION PILES. REACTION BEAM LENGTHS OF ABOUT 20 TO 22 FEET ARE NECESSARY TO ADEQUATELY MINIMIZE STRESS INFLUENCE OVERLAP BETWEEN REACTION AND TEST PILES FOR STRAIGHT PILES IN RELATIVELY HOMOGENEOUS SOILS. REACTION PILE SPACING WAS NOT CONSIDERED CRITICAL FOR THE SGC TESTS AT SITE A BECAUSE THE TEST PILES ARE END-BEARING AND THE BELLED REACTION PILES AT APPROXIMATELY THE SAME DEPTH TEND TO RELIEVE OVERBURDEN PRESSURE FROM THE STRESSED ZONE AND MAKE THE SETTLEMENTS SOMEWHAT CONSERVATIVE FOR THE DEPTH TESTED. HOWEVER, REACTION PILE SPACING IS IMPORTANT FOR TESTS ON THE CAF SOILS AT TEST SITES B AND C SO THE LOAD FRAME APPARATUS WAS DESIGNED ACCORDINGLY.

THOUSAND TON TESTS WERE SELECTED FOR THE PROGRAM TO MAKE THE TEST LOADS AS CLOSE AS POSSIBLE TO FULL-SCALE FOUNDATIONS FOR HEAVIER HIGHWAY STRUCTURES.

THE MOBILE LOAD FRAME DESIGNED AND FABRICATED FOR THE PROJECT CONSISTS OF AN ALL-WELDED A36 STEEL PLATE GIRDER APPROXIMATELY 6 FEET WIDE AND 25 FEET LONG. THE FRAME IS MOUNTED ON A HEAVY-DUTY DUAL AXLE TRAILER WHEEL ASSEMBLY WITH A KINGPIN AND PLATE ASSEMBLY IN FRONT TO RECEIVE THE FIFTH WHEEL OF A SEMI-TRACTOR RIG. THE FRAME WEIGHS APPROXIMATELY 26 TONS AND IS SUPPORTED BY FOUR HYDRAULIC RAM OUTRIGGERS WHEN NOT IN TOW BY THE SEMI-TRACTOR UNIT. PROVISIONS FOR SLIGHT ADJUSTMENT OF THE CROSS BEAMS WITH SMALL HYDRAULIC JACKS HAVE BEEN INCORPORATED INTO

THE DESIGN SO THAT THE UNIT CAN BE POSITIONED OVER THE TEST PILE AND REACTION PILES WITH CONSIDERABLE EASE.

ALSO INCORPORATED INTO THE DESIGN OF THE LOAD FRAME, IS THE CAPACITY TO UTILIZE A SINGLE REACTION PILE AT EACH END OF THE FRAME FOR APPLIED TEST LOADS UP TO 500 TONS. THIS DESIGN FEATURE PROVIDES VERSATILITY IN THE MOBILE LOAD FRAME SO IT CAN BE USED ECONOMICALLY AND EFFICIENTLY IN VIRTUALLY ANY LOAD TESTING PROGRAM.

THE STRUCTURAL DESIGN OF THE MOBILE LOAD FRAME, DEFINED BY THE REQUIREMENT TO IMPOSE A DOWNWARD LOAD OF 1000 TONS ON A TEST PILE, ALSO PERMITS AN UPLIFT LOAD CAPABILITY OF THE FRAME TO A MAGNITUDE OF 375 TONS. THE CAPACITY TO PERFORM UPLIFT TESTS ON A PILE OR ANCHOR INSTALLED IN A VERTICAL POSITION, IS ACCOMPLISHED BY ADEQUATELY BLOCKING EACH END OF THE LOAD FRAME, PLACING ANCHOR RODS THROUGH A PATTERN OF HOLES PROVIDED IN THE CENTER OF THE LOAD FRAME AND BY PLACING 1 OR 2 OF THE HYDRAULIC RAMS FROM THE JACKING MODULE ON TOP OF THE MOBILE FRAME.

THE LOAD FRAME IS MOBILIZED BY A 3 AXLE SEMI-TRACTOR RIG. WHEN THE LOAD FRAME IS IN POSITION, THE FRAME IS LEVELED WITH THE HYDRAULIC RAM OUTRIGGERS AND THE SEMI-TRACTOR RELEASED FOR THE REMAINDER OF THE TEST SET-UP AND LOAD TESTING PERIOD.

FOR THIS TESTING PROGRAM, THE LOAD FRAME WAS ANCHORED TO EACH REACTION PILE BY FOUR 1 3/8 INCH DIAMETER HIGH STRENGTH STRESSTEEL RODS (160,000 PSI). THE 4 ANCHOR RODS WERE THREADED INTO A 15 INCH SQUARE, 2 INCH THICK, HIGH STRENGTH STEEL PLATE HELD IN PLACE BY A TEMPLATE TO A POINT APPROXIMATELY 6 INCHES ABOVE THE BASE OF THE REACTION PILE. THE RODS WERE CAST IN THE CONCRETE ANCHOR PILES AND EXTENDED FROM THE

ANCHOR PLATE TO 3 INCHES BELOW GROUND ELEVATION. WHEN THE LOAD FRAME WAS IN POSITION, THE ANCHOR RODS WERE ATTACHED TO THE LOAD FRAME RODS BY MEANS OF A THREADED COUPLER. FIGURE 20 SHOWS THE LOAD FRAME IN THE LOAD TESTING POSITION WITH REACTION PILES ATTACHED. FIGURES 1 THROUGH 8 SHOW VARIOUS PHOTOGRAPHS OF THE LOAD FRAME AND AUXILIARY EQUIPMENT.

THE HYDRAULIC JACKING UNIT, CONSISTING OF FOUR 300 TON DOUBLE ACTING HYDRAULIC RAMS OPERATING IN SERIES, IS LOWERED FROM THE LOAD FRAME TO THE TOP OF THE PILE BY MEANS OF 2 HAND OPERATED RATCHET CABLE PULLERS. HYDRAULIC PRESSURE IS SUPPLIED TO THE HYDRAULIC RAM SYSTEM BY AN SC MODEL 600 AIR-HYDRAULIC PUMP. AIR PRESSURE TO ACTUATE THE HYDRAULIC PUMP IS SUPPLIED BY AN 85 CFM OR LARGER AIR COMPRESSOR AND FINELY CONTROLLED BY A DIAPHRAM VALVE.

THE FOUR HYDRAULIC RAMS WERE MANUFACTURED AND INDIVIDUALLY CALIBRATED FOR THIS PROJECT BY BAYOU INDUSTRIES OF CHANNELVIEW, TEXAS. A 20,000 PSI STAINLESS STEEL HEAVY-DUTY PRESSURE GAUGE MARKED IN INCREMENTS OF 200 PSI AND READABLE TO 100 PSI IS USED TO MEASURE PRESSURES OF THE CALIBRATED SYSTEM. A SYSTEM PRESSURE OF 13,100 PSI IS REQUIRED TO APPLY A 993 TON LOAD TO THE TOP OF THE TEST PILE.

DESIGN OF A TELLTALE INSTRUMENTATION SYSTEM FOR TEST SITES B & C

PREVIOUS STUDIES BY BARKER AND REESE (37) EVALUATED FIVE VARIOUS METHODS OF PILE INSTRUMENTATION FOR THE PURPOSE OF DEFINING THE DISTRIBUTION OF LOAD ALONG A DRILLED SHAFT BY MEASUREMENT OF STRAINS AT VARIOUS POINTS IN THE SHAFTS. THE NUMEROUS ADVANTAGES AND DISADVANTAGES OF EACH TYPE OF INSTRUMENTATION SYSTEM WERE EVALUATED WITH THE FOLLOWING CRITERIA NOTED.

1. An indirect mechanical telltale method of instrumentation would eliminate the high cost of sophisticated readout equipment required for electrical systems.

2. Strain gauge devices are highly sensitive to moisture penetration when buried in concrete and elaborate methods to keep moisture out of the devices must be employed.

Considering these factors, a telltale instrumentation system was selected for the CAF soils at Sites B and C where an evaluation of load transfer was necessary. Due to the inherent lateral and vertical variability of soils involved, it was desired to perform as large a number of tests as possible and to minimize instrumentation costs.

Design considerations for the mobile load frame placed limitations on a telltale method of pile instrumentation. A maximum distance of 8 inches from top of pile to ground level is necessary to permit adequate clearance between the jacking module and top of pile at the time the mobile frame is pulled into testing position by the semi-tractor rig. The four 300 ton hydraulic rams in the jacking module have a minimum base and top plate dimension of 24 inches square. Likewise, a total test load of 1000 tons applied to the pile surface required a plate thickness of 5 inches. Little or no area remains available on the pile surface and in the peripheral area above ground level to efficiently permit the protrusion of a telltale system.

A telltale type compressometer, shown on Figure 21, was developed for the study using standard pipe fittings and sections. Movement of the device is sensed by a linear voltage displacement transducer (L.V.D.T.) held securely in a housing by allen screws. The L.V.D.T. stylus rests on top

OF A STAINLESS STEEL WIRE HELD TAUT BY A 2 INCH LONG COMPRESSED SPRING. FIGURE 21 SHOWS THE ASSEMBLED UNIT IN POSITION IN THE TEST PILE. A FLUKE DIGITAL VOLTMETER OWNED BY THE ARIZONA HIGHWAY DEPARTMENT WAS USED FOR MICROVOLTAGE READOUT OF EACH L.V.D.T. COMPRESSOMETER. A TRANSFORMER-SWITCHING UNIT PROVIDED AN EXCITATION OF 24 VOLTS D.C. FROM A 110 VOLT A.C. POWER SOURCE AND A MULTIPLE POSITION SWITCH TO PERMIT A RAPID VOLTAGE READING FROM 1 TO 8 L.V.D.T. UNITS.

THE L.V.D.T. COMPRESSOMETERS WERE ASSEMBLED TO PREDETERMINED LENGTHS AND ATTACHED IN PAIRS (180 DEGREES APART) TO THE REINFORCING CAGE OF EACH TEST PILE. THE REINFORCING STEEL CAGE WAS FABRICATED FROM SPIRALLY ROLLED NO. 3 SMOOTH WIRE AND FOUR NO. 7 LONGITUDINAL REINFORCING STEEL RODS.

THE INSTRUMENTED CAGE WAS SUSPENDED FROM A TEMPLATE AT GROUND LEVEL AND SO CONSTRUCTED TO HOLD EACH L.V.D.T. COMPRESSOMETER TELLTALE IN POSITION DURING THE CONCRETE POUR. THE CONCRETE, WHEN POURED IN EACH TEST PILE, WAS TEMPORARILY STOPPED JUST BELOW THE 2 INCH P.V.C. PLASTIC CAP; AT WHICH TIME THE SPRING WAS COMPRESSED AND THE WIRE SECURED WITH THE SCREW CAP. THE 2 INCH P.V.C. PIPE WAS THEN INSERTED IN THE P.V.C. CAP, STUFFED FULL WITH RAGS AND THE REMAINDER OF THE CONCRETE POURED AND FINISHED. FOLLOWING THE INITIAL SET, AND WHILE THE CONCRETE WAS STILL GREEN, GROOVES WERE ETCHED IN THE CONCRETE TO PERMIT PASSAGE OF THE L.V.D.T. WIRE UNDER THE 5 INCH STEEL PLATE AT TIME OF LOAD TESTING. THIS PROCEDURE WORKED VERY WELL DURING CONSTRUCTION STAGES OF THE TEST PILES AND PROVIDED AN EFFICIENT L.V.D.T. ASSEMBLY AT THE TIME THE LOAD TESTS WERE PERFORMED. THE L.V.D.T. AND HOUSING UNITS WERE INSERTED AT THE TIME OF LOAD TESTING AND REMOVED FOLLOWING EACH LOAD TEST.

Test Piles & Reaction Piles

Seven load tests were performed at Site A. Test pile diameters were selected varying from 2.5 to 9.3 feet so the effect of width of loaded area could be investigated. All but the 2.5 foot diameter piles were belled. The shafts of the piles were separated from the soil by sonotube so that the tests were strictly a measure of end-bearing. A visible annular space was present between the sonotube and surrounding soil and no friction was encountered in placing the sonotube. The maximum diameter was selected so that the bearing pressure at 1000 tons (28.6 ksf) would be on the order of those which would be used in full-scale design. Test pile depths varied from 15.5 to 18.5 feet.

Ten test piles were performed at both Sites B and C. Straight piles, belled piles and piles with small multiple bells or "shear collars" were tested. Piles TPC-4 through TPC-6 were tested in end-bearing only by separating the shafts from the soil by means of sonotube. Voids were created beneath piles TPC-7 through TPC-10 by placing ice in the lower 2 feet of the excavations so that they could be tested in side shear only. The effectiveness of creating voids was verified by probing through small casings installed at time of test pile construction.

The configuration and details of the test piles for the three sites are shown schematically in Figures 22 through 26. The location of the test and reaction piles are shown in Figures 13 through 15.

Reaction pile capacity was analyzed by methods developed by Meyerhof and Adams (38). Belled piles, 7.0 feet bell diameter and 2.5 feet shaft diameter, were designed for each site. Reaction pile depths were about 16 feet at Site A, 20 feet at Site B and 16 feet at Site C.

Either 4 reaction piles in a 7 x 20 foot diameter or 2 reaction piles spaced 23'6" apart, depending upon the total test load, were used. Tables 1, 2 and 3 show the exact dimensions of the various test and reaction piles.

Concrete was a 5000 psi 28 day strength design placed and vibrated at a 3 to 5 inch slump (a 7.5 sack mix). Results of the concrete compression and slump tests are reported in Appendix C (pages C-105 through C-108).

Load Testing Procedures

The "maintained load" test procedure was used for all tests in this study with loading increments varying from about 23 tons to 60 tons. Each load increment was maintained for 30 minutes and in some tests, 60 minutes. However, test piles TPA-7 at a load increment of 689 tons (19.9 ksf end-bearing pressure), TPB-2 at a load increment of 263 tons and TPC-1 at a load increment of 567 tons were held for extended time periods of 1060 minutes, 990 minutes and 1320 minutes, respectively.

Digital voltmeter readings from the L.V.D.T. compressometer telltales were taken at 4 minutes and 19 minutes when 30 minute time increments were used and at 4, 19 and 49 minutes when 60 minute time increments were used on the various test piles at Sites B and C. Strain readings obtained are presented in Appendix C (pages C-109 through C-129).

Settlements, referenced to the ground surface, were measured with two 3 inch diameter face extensometer dials with a measurement range of 3 inches and sensitive to the nearest 0.001 inch. The dials were supported on 4 x 4 inch wood reference beams with steel pin reactions being 10.5 feet from the center of the test piles.

The load-settlement curves for the test piles at Sites A, B and C are given in Figures 27 through 34. Time-settlement curves for each load increment are shown in Figures 35 through 65. Long-term portions of the time-settlement curves for the extended time periods of TPA-7, TPB-2, TPB-5 and TPC-1 are shown in Figures 42, 45, 49 and 56.

The load and time settlement curves are corrected for compression in the concrete at test Site A and represent movement at the base of the piles. Total correction at maximum loads of 993 tons applied was about 0.13 inch. The reported settlements are believed to accurately reflect settlements at the ground surface of prototype piles as compression in the concrete will be very slight for normal working stresses. Settlement readings at test Sites B and C were not corrected for elastic compression in the concrete due to the lower total loads applied to most of the test piles.

CHAPTER VI - EVALUATION OF RESULTS - SITE A

General

The load tests at Site A were intended to investigate the performance of drilled piles under the range of bearing pressures likely to be used for actual foundations. The practical problem for the SGC soils was to evaluate the relationship between settlement, bearing pressure and width of foundation so that design curves could be developed to estimate settlements of full-scale foundations. It does not appear that providing an adequate factor of safety against shear failure will be a problem for drilled foundations bearing on the SGC soils for the range of bearing pressures, diameters and depths likely to be used in design.

Ultimate bearing pressures for the test piles were calculated by the semi-empirical Terzaghi method (2) which is intended for the analysis of shallow foundations. A range of angles of internal friction, Ø, of 38 to 44 degrees were selected for use in computations based on data presented by Leps (39) for coarse granular materials. Results of ultimate bearing capacity computations are given in Table 4. As can be seen, an ultimate bearing pressure of 294 kips/sq. ft. was calculated for TPA-1 using $\phi = 44^{\circ}$. An actual bearing pressure of 425 kips/sq. ft. was reached at maximum test load and it did not appear that failure had been defined. For the larger diameter piles, the maximum capability of the load frame did not permit the test to reach the levels of the computed values. However, the test for the 2'5" diameter pile (TPA-1) exceeded the computed ultimate value of the 9'5" diameter pile. It appears the Terzaghi method is well on the conservative side for the configuration of piles involved and can be safely used as a check of ultimate bearing capacity. More realistic evaluations of ultimate bearing capacity might be obtained by methods proposed by other investigators such as Meyerhof (40).

ARCHING PHENOMENON DISCUSSED BY VESIC (16) SHOULD BE CONSIDERED IN THE COMPUTATIONS FOR DEEPER DRILLED PILES.

EFFECT OF VARIATIONS IN SOIL CHARACTERISTICS

FOR PURPOSES OF ANALYSIS, SETTLEMENT INDICATED BY THE LOAD TESTS IN THIS STUDY FOR 20 KIPS PER SQUARE FOOT BEARING PRESSURE HAVE BEEN PLOTTED VERSUS BASE DIAMETER IN FIGURE 66. AS INDICATED, A DEGREE OF SCATTER OF DATA IS PRESENT. THE LARGER SETTLEMENT FOR TPA-7 PROBABLY IS DUE, IN PART, TO THE SAND LENSE PRESENT AT ABOUT 3.5 TO 5.0 FEET BELOW ITS BASE. HOWEVER, MUCH OF THE SCATTER IS UNDOUBTEDLY DUE TO VARIATIONS IN RELATIVE DENSITY. RESULTS OF SOME LARGE DIAMETER LABORATORY COMPRESSION TESTS (ONE-DIMENSIONAL CONSOLIDATION TESTS) PERFORMED BY KJAERNSLI AND SANDE (41) ON COARSE GRAVELS SIMILAR TO THE SGC ILLUSTRATE THE IMPORTANCE OF RELATIVE DENSITY. THESE TESTS INDICATED THAT LOOSE MATERIALS WERE ABOUT 50 PERCENT MORE COMPRESSIBLE THAN DENSE MATERIALS OF THE SAME GRADATION.

OWING TO THE LARGE DIAMETER OF THE PARTICLES INVOLVED, THE RELATIVE DENSITY OF THE SGC SOILS IS DIFFICULT TO DETERMINE. HOWEVER, THE SGC UNDOUBTEDLY HAS THE VARIATIONS IN RELATIVE DENSITY THAT ARE INEVITABLY PRESENT IN NATURALLY OCCURRING GRANULAR SOIL DEPOSITS.

RELATIVE DENSITY DETERMINATIONS IN THE SGC HAVE BEEN MADE AT A PLATE LOAD TEST SITE ABOUT 1.5 MILES EAST OF SITE A. THE TESTS WERE, IN PART, IN FINER INTERVALS OF THE DEPOSIT. LARGE VOLUME IN-PLACE DRY DENSITIES DETERMINED BY THE USE OF CALIBRATED SILICA SAND RANGED FROM 116 TO 140 POUNDS PER CUBIC FOOT. LABORATORY DENSITY DETERMINATIONS BY ASTM D2049-69 (33) INDICATED MAXIMUM DENSITIES BETWEEN 139 AND 144 AND MINIMUM DENSITIES BETWEEN 118 AND 122. RELATIVE DENSITIES RANGE FROM 50 TO 96 PERCENT.

Although these tests may not be indicative of the general characteristics of the deposit, they do illustrate variations in relative density. Although the SGC is relatively uniform across the site, small variations in gradation are, no doubt, partly responsible for variations in the settlement of individual test piles from an "average" curve. In general, the compressibility of granular soils increases with increasingly finer gradation. The data reported by Kjaernsli and Sande (41) involving hard, smooth, rounded particles up to about $2\frac{1}{2}$ inches in diameter, indicate that well graded materials are somewhat less compressible than poorly graded materials at the same relative density.

Evaluation of Becker Hammer Drill Data

As a part of the foundation investigations for 2 sections of elevated freeway, a total of 285 Becker Hammer Drill borings were drilled along a corridor beginning about $\frac{1}{2}$ mile west of test Site A and extending easterly for 3 miles. Borings penetrated the SGC about 30 to 75 feet. They provide a valuable body of information on the uniformity of the deposit.

Clay and clayey sand layers within the SGC extensive enough to require special consideration in design were encountered in 10 borings; looser sand layers extensive enough to require special consideration occurred in 56 cases. Some of these layers were near the contact of the SGC and could be avoided by extending foundations a few feet below the contact.

In order to evaluate how Becker Hammer Drill blow count, N_B, correlated with performance of the test piles, weighted N_B was determined for the test holes drilled immediately adjacent to the piles. Weighting was made on the basis of relative Boussinesq stress influence at the midpoint of each 6 inch layer to a depth of about 2 diameters below the base of piles. Calculation procedures were similar to those described by Coon and Merritt (42) for the correlation of diamond core recovery of rock to modulus of deformation. The following results were obtained.

Test Pile	Blows Per Foot: Weighted N_B	Blows Per Foot: Mean N_B	Difference
TPA-1	50.10	47.12	2.98
TPA-2	37.86	38.00	0.14
TPA-3	49.56	47.74	1.82
TPA-4	28.32	29.14	0.82
TPA-5	50.96	49.58	1.38
TPA-6	45.56	44.76	0.80
TPA-7	37.02	41.16	4.14

The weighted N_B values varied by no more than 4.14 blows per foot from mean values for the same range of depth. Modulus of deformation of the SGC was computed for each test pile at various bearing pressures by the theory of elasticity. Procedures outlined by Bowles (43) were used as follows:

$$E = \frac{qD\,(1-u^2)\,I_W}{S} \quad \text{. . . . . (Equation 1)}$$

Where:
- E = modulus of deformation
- q = bearing pressure
- D = diameter of pile
- S = settlement of pile
- u = Poisson's ratio
- I_W = influence factor (for end-bearing only)

An influence coefficient, I_W, of 0.88 was used for a rigid circular foundation. Void ratios for the SGC appear to range from about 0.2 to 0.4 and it is believed that the average

VALUE IS NEAR THE LOWER FIGURE. THUS, $u = 0.15$ WAS ASSUMED IN CALCULATIONS.

BECAUSE GRANULAR SOILS DO NOT BEHAVE ELASTICALLY, AND IN NATURE ARE NOT COMPLETELY HOMOGENEOUS OR ISOTROPIC, THE CALCULATED MODULUS E IS AN APPROXIMATION OF THE AVERAGE MODULUS OF DEFORMATION OF A ZONE OF SOIL WHICH IS SIGNIFICANTLY STRESSED.

THE RELATIONSHIP BETWEEN E AND Q FOR EACH TEST PILE IS SHOWN ON FIGURE 67. FIGURE 17 SHOWS THE RELATIONSHIP BETWEEN E AND WEIGHTED N_B FOR VARIOUS BEARING PRESSURES. WEIGHTED N_B VERSUS E AND Q VERSUS E FOR TEN 30 INCH PLATE LOAD TESTS PERFORMED AT VARIOUS DEPTHS IN THE BOTTOM OF TWO LARGE DIAMETER BORINGS ABOUT 1.5 MILES EAST OF SITE A ARE ALSO SHOWN ON FIGURES 67 AND 17. AS CAN BE SEEN, E INCREASES IN A GENERAL WAY WITH INCREASING N_B.

DURING THE COURSE OF EXPLORATORY DRILLING, STANDARD PENETRATION TESTS WERE TAKEN THROUGH THE BECKER DRIVE PIPE IN A NUMBER OF CASES WHERE FINE ENOUGH GRANULAR INTERVALS WERE PRESENT SO THE SPLIT BARREL SAMPLER COULD BE DRIVEN AT LEAST A FOOT. BECKER BLOW COUNT, N_B, IS PLOTTED AGAINST STANDARD PENETRATION RESISTANCE, N, FOR THESE TESTS IN FIGURE 18. THE STANDARD PENETRATION TESTS WERE UNDOUBTEDLY AFFECTED TO SOME DEGREE BY THE VIBRATIONS OF THE BECKER DRILLING. HOWEVER, THEY INDICATE A VERY GENERAL RELATIONSHIP BETWEEN N_B AND N. SIMILAR DATA DEVELOPED BY WALKER (44) ALSO IS SHOWN ON FIGURE 18. THIS DATA SUGGESTS THAT CAREFULLY PERFORMED AND ANALYZED BECKER DRILLING PRODUCES A DYNAMIC PENETRATION RESISTANCE AS MEANINGFUL AS THE STANDARD PENETRATION TEST. ALSO INCLUDED IN FIGURE 18 IS THE "BEST FITTED" CURVE CONSTRUCTED BY REGRESSION ANALYSIS.

FROM THE DATA DISCUSSED ABOVE, IT IS BELIEVED THAT N_B INDICATES GENERALLY THE DEGREE OF VARIABILITY OF COMPRESSIBILITY OF THE SGC. NORMAL DISTRIBUTION OF MEAN N_B IN THE UPPER 20

FEET OF THE SGC FOR THE TWO FOUNDATION INVESTIGATIONS INVOLVING 155 AND 130 BORINGS AND THE MEAN N_B AT SITE A WAS ASSUMED. GRAPHIC REPRESENTATION OF FREQUENCY OF BLOW COUNT OCCURRENCE FOR THE ABOVE MENTIONED THREE SETS OF BECKER BLOW COUNT DATA IS SHOWN ON FIGURE 68. THE WIDE RANGE OF DISTRIBUTION FOR UNIT II IS BELIEVED TO BE DUE TO THE FACT THAT ONE OF THE TWO BECKER HAMMER DRILLS USED WAS FOUND TO BE DELIVERING MUCH LESS THAN 8000 FOOT POUNDS PER BLOW HAMMER ENERGY. THIS WAS CORRECTED ABOUT HALFWAY THROUGH THE DRILLING PROGRAM. HOWEVER, AS FIGURE 68 INDICATES, THE MEAN N_B FOR SITE A IS CONSIDERABLY LOWER THAN INDICATED FOR THE DEPOSIT AS A WHOLE AND ITS RANGE OF DISTRIBUTION COVERS THE LOWER RANGE TO BE EXPECTED IN THE DEPOSIT.

THE VARIATION OF MEAN N_B FROM 29 TO 49 OVER TEST SITE A, 30 x 80 FEET IN PLAN DIMENSION, ILLUSTRATES THE EXTREME DEGREE OF LATERAL VARIATION WITHIN THE DEPOSIT.

EFFECT OF WIDTH OF LOADED AREA

THE MODULUS OF DEFORMATION OF GRANULAR SOILS INCREASES WITH DEPTH. THUS, FOR A GIVEN BEARING PRESSURE, SETTLEMENTS OF FOUNDATIONS ON RELATIVELY HOMOGENEOUS, CLEAN, COHESIONLESS, GRANULAR SOILS INCREASE WITH INCREASING WIDTH OF LOADED AREA AT A DECREASING RATE. THE FACTORS INFLUENCING THE COMPRESSIBILITY OF GRANULAR SOILS AND THEIR VARIATIONS ARE DISCUSSED IN DETAIL BY BURMISTER (45, 46).

FOR PURPOSES OF ANALYSIS OF THE EFFECT OF FOUNDATION WIDTHS ON SETTLEMENTS, A POINT OF S = 0.2 INCH AND D = 6.0 FEET WAS ASSUMED ON FIGURE 66 FOR Q = 20 KIPS/SQ. FT. THIS POINT IS CONSISTENT WITH THE PERFORMANCE OF TPA-3, TPA-4, TPA-5 AND TPA-6 AND WAS THOUGHT TO BE NEAR AN "AVERAGE" CURVE FOR THE deposit. A modulus of deformation, E, of 29,700 psi was computed for this point. Settlements were then calculated for other widths on a curve by Equation 1; using the "average point" method suggested by Lambe and Whitman (47). In this method, the "average point" of stress within the bulb of significant stress influence is considered to be 0.75 D below the base of footing. E can then be varied as a function of the initial vertical confining pressure P_z at the "average point" as D changes. Curves are plotted on Figure 66 computed on the assumptions of E varying directly with P_z and E varying with the square root of P_z. A curve is also shown constructed on the assumption that E is constant with depth.

A number of investigators (46, 48, 49, 50, 51, 52) have reported case histories where the relationship between foundation width and settlement for a given bearing pressure was developed by field measurements. Several curves constructed from this data are plotted on Figure 66 to compare with the calculated curves.

Both Terzaghi and Peck (48) and Bjerrum and Eggestad (49) have presented data on the relationship between the settlement of small square plates and foundations of various widths. The Bjerrum and Eggestad (49) curves are based upon the evaluation of 14 case histories where detailed information on the soils, plate bearing tests and measurements of settlements of full-scale foundations were available.

Burmister (46) reports on a case of a 65 foot wide reactor foundation bearing on well graded sand with some gravel where small diameter plate bearing tests were performed in preliminary design studies.

Moorhouse and Sheehan (50) have summarized information on

THE SETTLEMENT-WIDTH RELATIONSHIP FOR PILE GROUPS INCLUDING STUDIES BY SKEMPTON, ET AL (51) AND MEYERHOF (52). BECAUSE MUCH OF THE DATA USED IN DEVELOPING THESE CURVES INVOLVED DRIVEN PILES, THEY ARE BELIEVED TO BE INFLUENCED BY DENSIFICATION OF THE SANDS BELOW THE PILE TIPS DURING DRIVING; PARTICULARLY FOR SMALLER WIDTHS.

DVORAK (53) REPORTS THE RESULTS OF BEARING TESTS WITH 0.8 AND 1.2 FOOT DIAMETER CIRCULAR PLATES AND A 2.3 FOOT SQUARE PLATE ON WELL GRADED SAND AND GRAVEL UP TO ABOUT 2.5 TO 4.0 INCHES IN DIAMETER WHICH APPEAR SIMILAR TO THE SGC. THE SETTLEMENTS INDICATED BY THESE TESTS AT 20 KSF ALSO ARE PLOTTED ON FIGURE 66 FOR COMPARISON ALONG WITH THOSE FROM THE 30 INCH PLATE BEARING TESTS PERFORMED ON THE SGC.

IN FURTHER EVALUATION OF THE BJERRUM AND EGGESTAD (49) DATA, MEIGH (54) STATES THAT SETTLEMENTS FOR COARSE, WELL GRADED MATERIALS FELL IN THE LOWER SECTOR OF THEIR LIMITS WHILE SETTLEMENTS FOR FINE, POORLY GRADED MATERIALS FELL WITHIN THE UPPER SECTOR. THE SGC, OF COURSE, FALLS IN THE CATEGORY OF COARSE, WELL GRADED MATERIALS.

FROM EVALUATION OF THE VARIOUS CASE HISTORY DATA, IT APPEARS THE MOST LIKELY "AVERAGE CURVE" FOR THE SGC IS NEAR THE CURVE BASED UPON E VARYING DIRECTLY WITH THE SQUARE ROOT OF P_z. THE CURVE, BASED UPON E VARYING DIRECTLY WITH P_z, SEEMS TO BE THE UPPER LIMIT OF WHERE AN "AVERAGE CURVE" COULD REASONABLY BE EXPECTED TO FALL. "AVERAGE CURVES" CONSTRUCTED ON THIS BASIS ARE PRESENTED IN FIGURE 77 FOR Q = 10, 15, 20, 25 AND 30 KIPS/SQ. FT.

EFFECT OF TIME

THE LONG-TERM MAINTAINED LOAD ON TPA-7 SHOWED THAT SETTLEMENT

WAS COMPLETE AT ABOUT 11 HOURS. THIS LOAD WAS MAINTAINED AT 19.9 KIPS PER SQUARE FOOT WHICH IS EXPECTED TO BE IN THE GENERAL RANGE OF BEARING PRESSURES WHICH WILL BE USED IN THE DESIGN OF FOUNDATIONS FOR VERY HIGH TOTAL LOADS ON THE SGC DEPOSIT. ULTIMATE SETTLEMENT WAS 19 PERCENT HIGHER THAN THE SETTLEMENT AT 30 MINUTES. SIMILAR TIME-SETTLEMENT RELATIONSHIPS WERE INDICATED BY THE 30 INCH DIAMETER PLATE BEARING TESTS. THIS RELATIVELY RAPID TIME-RATE OF SETTLEMENT RELATIONSHIP IS TYPICAL OF MUCH PREVIOUS DATA REPORTED FOR FOUNDATIONS ON GRANULAR SOILS.